

A Brief Review of Geotechnical Aspects and Monitoring of Gateway Arterial Roadworks North of the Brisbane River

A. LITWINOWICZ

Acting Engineer, Main Roads Department, Queensland

I.K. SMITH

Engineer/Geomorphologist, Hollingsworth Consultants, Brisbane

SUMMARY This paper provides a brief summary of geotechnical aspects of 24km of arterial road north of the Gateway Bridge. The terrain encountered includes 17km of swampland with soft soils extending up to 20m in depth. The main problems resulting from these conditions are those of embankment settlement and stability and the need for staged construction. The geotechnical design approach has involved observing embankment performance to confirm preliminary estimates and to permit construction programming. Several ground improvement techniques have been employed, which included preloading, surcharging, vertical drains, stone columns and geotextile and steel mesh reinforcement of embankments. Comparisons of predicted and actual settlement behaviour are made together with discussion of observed creep. Conclusions are drawn on the effectiveness of the techniques employed.

1 INTRODUCTION

The Gateway arterial comprises some 21km of roadway linking the Gateway Bridge Crossing of the Brisbane River to the Bruce Highway on the northern outskirts of the city (Figure 1). Several feeder connections occur along the route including the important East-West Arterial connection to Sandgate Road, in the vicinity of the Brisbane Airport. The alignments of both the Gateway and East-West arterials traverse significant extents of contemporaneous estuarine and marine deposits consisting of very soft silts and clays. Significant soft soil problems were envisaged at Schulz Canal, Pound and Nundah Creeks, and at several other locations indicated in Figure 1 but not covered here.

deformation, became clearly defined only as the project was progressing. Main Roads Queensland (MRQ) has for some years employed the Observational Method to settlement management, Robertson (1984). The method facilitates decisions regarding construction activities which must be made in sympathy with observed settlement and stability behaviour.

In order to manage and/or overcome the geotechnical problems within the tight time constraints imposed, several techniques for foundation improvement were employed, and generally achieved the construction and performance criteria which has been pre-determined. Since foundation deformation had the greatest bearing on the successful completion of the Gateway and East-West Arterials, this paper mainly addresses the difficult task of settlement prediction, design of appropriate treatments and comparison with actual performance. Embankment stability was of concern at several locations and a suitable discussion of stability is included. Overall, the techniques employed ensured successful construction and completion of two significant roadwork projects.

2 SITE STRATIGRAPHY

The subsurface conditions pertaining to some total 24km of roadworks is both complex and variable, and it is not intended to present such information in detail. In general, the overall area of interest is underlain by Quaternary sediments which are transitional between alluvial and coastal deposits, with the former being derived from existing creeks and occurring as a veneer on coastal sediments.

Soft, normally to lightly over-consolidated clays (OCR1-2) persist to varying depths of up to 20m and are often overlain by a thin desiccated crustal layer up to 1.5m thickness. At several locations, larger streams have incised the underlying tertiary formations at a time of lower sea level, and have since backfilled with marine clay with rising sea level. Sand drainage layers within the soft alluvium are often evident but discontinuous, with the absence of such layers in some areas.

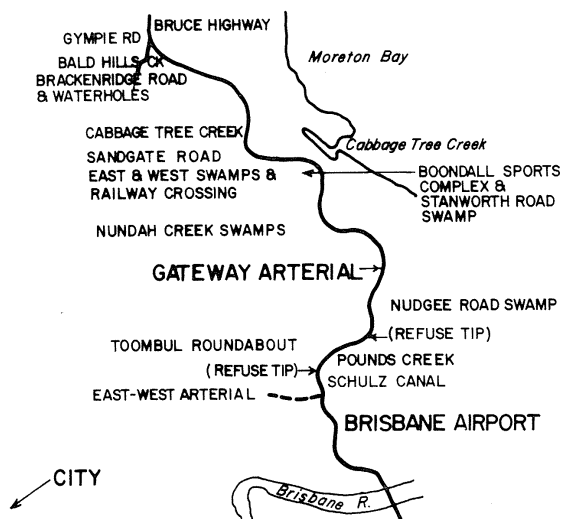


Figure 1 East-West & Gateway Arterial Roads

Design and construction of the Gateway Arterial project was managed using the "fast track" approach and thus some of the geotechnical problems, particularly regarding foundation

Figure 2 depicts typical subsurface conditions at Schulz Canal and Pound Creek.

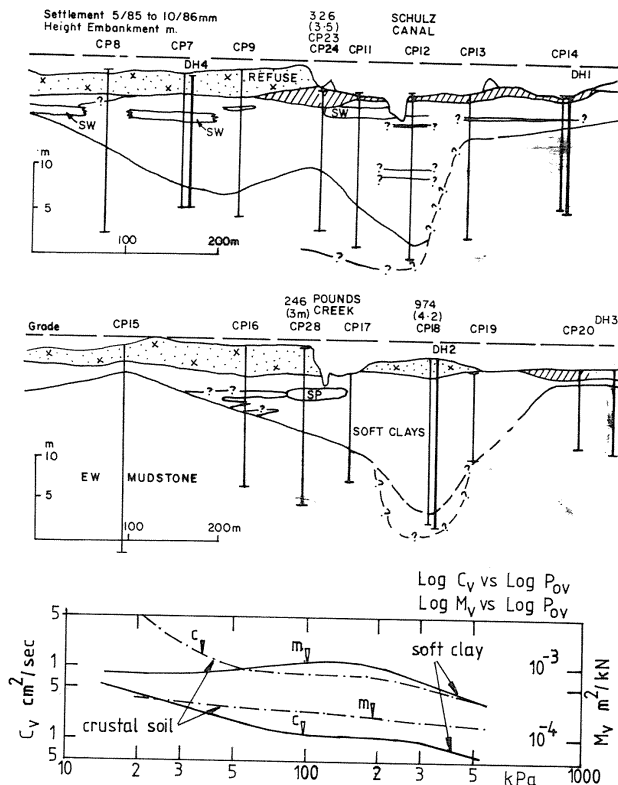


Figure 2 Schulz Canal and Pounds Creek

Note: Dark grey very soft to firm clay shown unshaded. CP = Cone Penetrometer DH = Drill Hole. Small Crustal layers of firm to stiff clay show hatched. At abutments CP 28 and circa CP 28 refuse was removed prior to building embankment.

3 GEOTECHNICAL INVESTIGATIONS

3.1 Field Activities

The field investigation for the Gateway Arterial project comprised 65 boreholes (together with undisturbed tube sampling), 75 CPT (Static Friction Cone Penetration Tests), 50 test pits and a considerable number of DC (Dynamic Cone) and FSV (Field Shear Vane) tests. Pressuremeter tests were conducted principally for the bridge sites, some cuttings and the dynamic compaction of refuse. This resulted in about one test site every 100m in the deeper swamps, ranging to a test site every 400 to 500m elsewhere.

A similar exercise had previously been carried out for the East-West Arterial, but on a condensed scale given the short (approx. 3km) length of the roadway. The CPT was considered the most useful tool since it provided rapid data gathering and a continuous sounding profile to identify potential sand drainage layers for settlement prediction.

The application of rod corrections to FSV results should be used with care where stiffer soils overlie softer ones. It was often found that the rod correction exceeded the vane shear strength

and this was resolved by preboring through the stiff soils or testing from cased boreholes.

3.2 Laboratory Testing

Standard laboratory test procedures for soft soils were carried out, and for both projects comprised the following:

- Classification testing (grading and plasticity).
- Oedometer testing with both vertical and radial drainage.
- Triaxial strength testing; mainly quick undrained with fewer consolidated undrained.

Typical material properties for the soft soils encountered along the alignments are presented in Table 1. The majority of soft soils were normally consolidated or lightly consolidated ($OCR < 2$). Typical representative M_v and C_v vs effective stress relationships for the soft clay in the vicinity of Schulz Canal are presented in Figure 2. The compressibility properties were assumed to be log-normally distributed at constant effective stress and average M_v values and 90% Highest C_v values were used in the analysis based on previous experience with soft alluvial deposits in South-East Queensland, Robertson (1984). Oedometer tests from the East-West Arterial generally indicated a ratio of horizontal to vertical permeability of 10.

TABLE I

TYPICAL SOIL PARAMETERS FOR SOFT SOILS NORTH OF THE BRISBANE RIVER

Location	LL	PL	M/C	Cu	S	γ	
GATEWAY ARTERIAL							
Serpentine (Schulz)	100	16	88*	10-15	3	1.57	
Nundah Ck	41-	31*	25*	34*	10-15	3	1.05
Sandgate Road	75	36	70	13	3-7	-	
Cabbage Tree Ck	80	26	80	13	3-4	-	
Bald Hills	102	14	102	3-10	3-5	0.70	
	71*	21*	31*			1.50	
Gympie Road	70	20	40	5-10	3	-	
EAST-WEST ARTERIAL	70	25	75	10-15	-	1.57	

* not the softest soils.

4 APPROACH TO MANAGEMENT OF GEOTECHNICAL PROBLEMS

4.1 Discussion of Geotechnical Techniques

4.1.1 General

The range of geotechnical techniques used to both improve stability and achieve the required settlement criteria were:

- (i) Preloading
- (ii) Surcharging
- (iii) Stability Berms
- (iv) Removal and Replacement of Unsuitable Material
- (v) Vertical Drains
- (vi) Stone Columns
- (vii) Geotextile and Steel Mesh Reinforcement
- (viii) Dynamic Compaction of Refuse

4.1.2 East-West Arterial

Construction of this section commenced in February 1985 with the installation of vertical drains at a spacing of 1.6 - 2.1m (triangular pattern). Placement of the 4m high embankment plus a 1m surcharge was completed by the end of July 1985. The surcharge remained in place for approximately 2 months at which time it was removed to facilitate the commencement of paving operations.

4.1.3 Gateway Arterial

For the Gateway Arterial preloading consisted of early construction of the embankment to its normal height with monitoring of settlements and pore pressures. At sites where the required in service settlement of 100mm (or 50mm at bridge abutments) could not be achieved in the 6 month time period stipulated, surcharging was employed. Preloading/Surcharging was employed for over 60% of the route and staged construction over about 10%.

At various locations, surcharges ranged from 0.5 to 2.0m height, vertical drain spacing from 1.4 to 2.8m (triangular pattern) and stone columns of 1.0m diameter were spaced at intervals of from 2.0m to 3.0m. Monitoring showed these measures to be highly effective in reducing settlement. The construction periods ranged from 2 to 9 months with individual surcharges often being in place for 3 to 8 weeks.

One of the main problems to be considered was the discontinuity between embankments and structures, especially where water channels increased the height of the embankments by several metres. At these sites and elsewhere where extremely soft muds exist, berms were employed together with the use of steel mesh F62 to F918, geotextile reinforcement and stone columns. At some of the bridge sites horizontal strains were of concern and these measures were again employed to reduce these to tolerable limits. To facilitate compaction of fills and to permit the passage of construction traffic, needle punched geotextiles were used at most soft soil areas although were omitted where a strong grass mat existed.

4.2 Settlement Analyses

Pre-construction analyses were carried out for suitably developed models using conventional one dimensional consolidation theory assuming elastic stress distributions, with the coefficient of volume compressibility (M_v) being obtained from oedometer tests or CPT soundings. Simplifying assumptions were made with regard to loading history and the selection and degree of reliance on drainage paths. The analyses however provided the means by which to programme construction activities to be later controlled by the outcome of intensive monitoring.

Vertical drains were designed using conventional radial drainage theory (eg. Parry et al 1985 and Barron 1948) with smear effects allowed for by the method of Richart (1959). This was performed by deriving a reduced effective drain well diameter based on a smeared zone of 1.2 times the diameter of the well, with a reduced permeability in the smeared zone 1/5th the horizontal permeability. The smear parameters were adopted based on previous experience, Robertson and Semple (1984).

Strengthening by way of stone columns was evaluated by imposing equivalent strips of high strength ($\phi=45^\circ$) material into conventional stability analyses. Reductions in deformation (both vertical and horizontal) were computed using the charts of Balaam and Booker (1985). Reinforcement using steel mesh was assessed by imposing a horizontal force equal to the ultimate tensile strength of the main bars. The purpose of the mesh was to improve short-term stability and thus corrosion was of little concern.

Pre-completion analyses were performed after suitable observation of field behaviour and generally took the form of graphical interpretation of settlement/time data as proposed by Asaoka and Suzuki (1979). The pre-completion analyses were aimed at predicting future outcomes of critical concern to construction programming such as:

- magnitude and time of final settlement
- time for removal of surcharge
- effectiveness of acceleration techniques
- need for additional treatment
- time required for a pre-determined in-service settlement to be satisfied.

Whilst 80-90% of consolidation is required for reliable future predictions from back analyses, the Asoka method offers the advantage of giving reliable predictions after only 60% of consolidation.

Immediate settlements were not generally calculated, as these occur during the construction period and were of no consequence to contract administration. Settlement predictions were for primary consolidation only and an arbitrary value of 10% of the primary consolidation was generally added for secondary settlement to arrive at a value for in-service settlement. It will be seen that such an approach to creep can significantly underestimate such settlements in some circumstances.

4.3 Stability Analyses

4.3.1 Analytical approach

Stability analyses were generally carried out for both circular and non-circular failure surfaces using conventional limiting equilibrium methods. Undrained analyses were carried out for the critical end of construction condition with undrained shear strengths being determined from triaxial testing, rod corrected vane results and inferred from CPT data as $q_u/18$. Where staging was concerned, strength gain was assessed by CPT or FSV at appropriate times.

Direct comparisons, from adjacent test locations, between FSV and CPT results generally supported a factor of 15-18 for obtaining c_u in very soft to soft soils and 13 in firm to stiff soils.

4.3.2 Discussion of factors of safety

Statements on embankment stability may be made either qualitatively or quantitatively. The former method has been employed extensively by geomorphologists, Brunsdon and Prior (1984); Crozier and Eyles (1980); and the latter by engineers, Gray and Leiser (1982); Schuster and Krizek (1978); Seigel (1983). However with respect to most slope stability problems it is recognised that there is little to choose between

either method with respect to accuracy, Smith (1986); Parry (1971). Hence it is important to recognise that quantitative statements as to factor of safety (F/S) are subjective.

Generally in engineering the quantitative approach is adopted with Factors of Safety (F/S) >1.5. For the Gateway Arterial appropriate F/S of between 1.6 and 1.9 were adopted for end of construction conditions and for use with undrained shear strengths derived from FSV and CPT testing. With time these F/S should increase to 2.4 due to consolidation and strength gain. The selection of these factors of safety has been described in detail by Smith (1987) and only a few examples of failures that have occurred at high F/S are quoted here. For a $\phi=0$ analysis, Parry (1971) lists a number of cases where failures have occurred in marine muds at high F/S, as shown in Table II. For the case of permanent works where life and property are in danger the margin for error could be increased to 50% and hence a F/S of $1.6 \times 1.5 = 2.4$ might be adopted. With intensive retesting and monitoring of pore pressures and prevailing shear strength, this might be reduced, Smith (1987).

TABLE II
FACTORS OF SAFETY AT FAILURE &
SOIL PARAMETERS FOR SOFT SOILS

CASE HISTORY	CONDITIONS	LL	PL	M/C	Cu	S	F/S
Peterson et. al. (1957, 1960)	Dykes Canada	85	26	45	20	1-2	1.5
Parry, McLeod (1967)	Launceston Australia	145	40	170	10	-	1.6
Eide (1967, 1968)	Bangkok Thailand	100	40	80	10	4-8	1.5
Foundation Engineering Ltd.	Escravos Mole Nigeria	29-109	16-41	27-107	10	8	1.3
Casagrande (1960)	Great Salt	73	31	-	18	-	2.0
Marsland (1957)	Thames Banks	90-150	30-70	-	17	4	1.4

It may be noted that F/S's as low as 1.3 and 1.2 are used in slope stability (Hong Kong, Geotechnical Manual for Slopes 1981). Whilst inspection of the conditions for use of these values, or any other references with low factor of safety, shows that these are inapplicable for soft soils and total stress methods, it is often the case that the assessment of risk is not in the hands of the geotechnical engineer, and other influences (eg. critical path for the project) may well control what is an acceptable risk. Definition of acceptable risk is the difficult task.

It should be noted that selection of an appropriate F/S is subjective and dependent upon the quality and quantity of material testing (both fill and soft soils), the usage of total or effective stress analyses and circular or

non-circular failure surfaces. In some special cases where lower bound strength values are used usually together with effective stress analytical methods, lower F/S have been adopted for the end of construction.

However in reality a higher F/S often operates due to strength gain during fill placement and the usual finding that consolidation occurs more rapidly than predicted. Due to the fast-tracking of Gateway Arterial, mainly undrained strengths were available and hence appropriate F/S's of 1.6 - 1.9 were adopted.

4.4 Instrumentation and Monitoring

Typical instrumentation included settlement plates, water overflow settlement cells (Main Roads Design), pneumatic piezometers, inclinometers and several banks of temporary movement stakes. Settlement plates were frequently damaged by construction traffic whereas only 2 of the cells were temporarily affected when lines were severed. The piezometers installed included both standard high and low air entry tips and microtips. Over the two year monitoring period no apparent malfunctions were recorded with any of the piezometers and hence, on a cost basis, and for ease of installation the microtips are to be recommended.

Monitoring of the Gateway Arterial commenced during late 1985 and was continued through to construction completion in September 1986 and beyond. At East-West Arterial monitoring commenced in early 1985 and, due to significant creep, is ongoing.

5 COMPARISON OF PREDICTED VERSUS OBSERVED BEHAVIOUR

Rather than discuss data obtained from detailed monitoring of 24km of roadworks, this section is devoted to presenting some of the more interesting findings of significance. The specific locations discussed are:-

- (i) East-West Arterial @ Ch.1200 - Significant Creep Settlements
- (ii) Gateway Arterial @ Schulz Canal - Performance of Vertical Drains
- (iii) Gateway Arterial @ Nundah Creek - Performance of Stone Columns.

5.1 East-West Arterial

For this project foundation treatment consisted of installing vertical drains on a 1.6m triangular grid. The embankment was constructed over a period of several weeks to a height of 5m approximately including a 1m surcharge. Monitoring indicated that settlements were greatest at Ch.1200 (as expected). The Asaoka plot of settlement history is given in Figure 3 whilst Table III provides a comparison with predicted behaviour.

The surcharge was removed after 3 months since it was reasonably estimated from settlement trends that primary consolidation had been achieved for the at-grade embankment. Whilst it was evident that some creep would occur, the ensuing 500mm of settlement over 2 years was unexpected. The divergence of the Asaoka plot would indicate that the settlement is creep orientated. Post-construction testing of the soft clay indicates that the current rate of creep is 5

times that predicted by oedometer testing (Reeves, 1987). The performance witnessed also indicates the need (and difficulty) of interpreting oedometer data particularly at or near the pre-consolidated pressure, Simons (1987). Coupled with the significant vertical settlement some 250mm of lateral bulging has also been recorded by inclinometer.

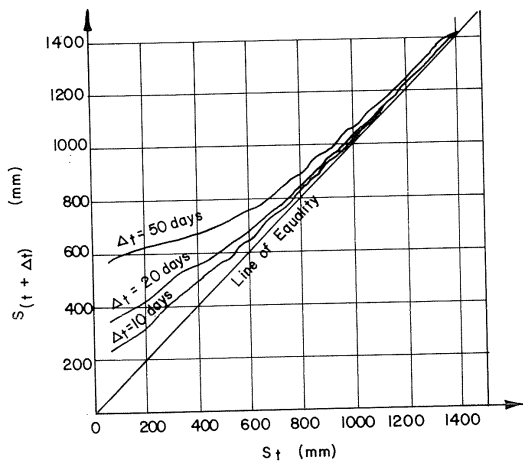


Figure 3 East-West Arterial: Settlement at Ch.1200 SG3 Asoaka Plot

TABLE III

EAST-WEST ARTERIAL: SETTLEMENT @ CH.1200

	Settlement (mm)		T90 (Days)	
	Predicted	Asaoka	Predicted	Actual
With 1m Surcharge	1000	950		100
Without Surcharge	750	750	180	

Whilst this behaviour is extraordinary, it is by no means unique. Brawner and Lea (1963) observed creep rates up to 4 times those predicted from oedometer tests. Walker (1969) indicated that shear creep rate is a function of the applied stress ratio whilst volumetric creep is independent of the applied stress. Oedometer tests would therefore be unsuitable for reliable creep determination as evidenced by the often constant C_α once the pre-consolidation pressure has been exceeded.

Another interesting feature at this location is the very slow rate of dissipation of excess pore pressures, although primary compression is thought to have ceased (see Figure 4). Mitchell (1984) discusses this phenomenon and attributes it to pore pressures generated as the soil is restructured ("de-structured") during secondary compression thus adding a component to the decaying primary pore pressures. Perhaps the accelerated rate of creep is due to the partial dissipation of such pore pressures more quickly than "normal" due to the occurrence of wick

drains. That is, excess pore pressures may not have decayed at all without drains.

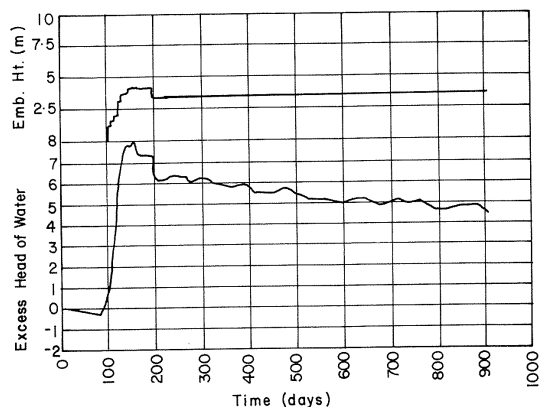


Figure 4 East-West Arterial: Piezometer No. 4

At the present rate of creep, a further 1m of settlement will occur in the next 25 years resulting in 2m of settlement for a 4m high embankment. However, the functioning of the roadway has not yet been affected except for pondage of water against the centre median, which will be corrected by installation of a median drain.

5.2 Gateway Arterial

5.2.1 Schulz Canal/Pound Creek

Settlement behaviour in this area was intensively monitored at 3 locations viz at CPT's 18, 24 and 28 to determine the effectiveness of vertical drains at 1.4m spacing. Settlements were initially predicted using oedometer data, however a comparison was also made based on correlations of m with q_c as proposed by Bachelier and Parez (1965), and using the method of Buisman (1936). The results for settlement magnitude and rate are given in Table IV.

TABLE IV

SETTLEMENT BEHAVIOUR AT SCHULZ CANAL/ POUND CREEK

Loca- tion	Height	Settlement (mm)		T90 (Days)		
		Oedo- meter	Bachelier (Buisman)	Actual	Pre- dicted	
CP18	(4.2m)	1100	1090(640)	970	180	190
CP23 & 24	(3.5m)	500	340(270)	330-450	160	120-175
CP28	(3.0m)	300	220(130)	250	120	150

It can be seen from the results of the monitoring that both magnitude and rate of consolidation were predicted quite closely using conventional laboratory procedures, although previous experience was exercised in the statistical selection of material properties. It is also evident that the smear properties adopted were

reasonable on the basis of the observed performance. The following characteristics of installation effects are recommended for closed mandrel driven wick drains.

Diameter of Smear Zone/Well diameter = 1.2.
Ratio of horizontal permeability to smear permeability = 5.

The simpler methods of estimation of settlement magnitude using CPT data correlated to M_v appear to give reasonable comparisons. This data is far simpler and more expedient to obtain and warrants consideration for estimates of settlement magnitude. A conversion factor of $m_v = 1+5q_c$ was used which overall at this site underestimated settlements by 3%. The correlations would need to be calibrated for a wide variety of soft soils before they could be used with confidence at other locations.

5.2.2 Nundah Creek

At the western approach to Nundah Creek, stone columns 1m ϕ were installed at 3m centres together with steel mesh to improve stability in an area where very soft to soft clays ranged from 3 to 4m depth. Conventional undrained analyses were undertaken by including the stone columns as equivalent strips of $\phi = 45^\circ$ material. The analyses indicated an end of construction F/S of 1.7 for the 2.5m high embankment. Installation of the foundation reinforcement was carried out without problem, and the performance of the embankment to the present has been excellent with no sign of tension cracking or other distress.

The method of Balaam and Booker (1985) was used to correct for settlements in the stone column treated area. Settlements both within and outside the treated area are given in Table V. It is clear that the rate of consolidation is faster in the area of columns with T90 being achieved in 35 days compared with 60 days in untreated areas. The magnitude of settlement is also some 50% of that in the untreated area. For similar circumstances reported by Waterton and Foulsham (1984) similar results have been achieved viz T90 < 100 days and settlement of 100mm. It should be noted that little creep consolidation has been witnessed to date. There was also considerable doubt as to the correct value of M_v .

TABLE V
SETTLEMENT BEHAVIOUR AT NUNDAH CREEK
- WESTERN APPROACH

Loca- tion	Settlement (mm)			T90 (Days)	
	Oedometer	Bachelier	Actual	Pre- dicted	Actual
* SC2	110-160	120	100	100	35
SC3	150-230	290	230)	
SC5	140-170	190	250) 180	50
SC7	180-260	190	210)	60

nb. SC = Settlement Cell

* Within area treated by Stone Columns.

The CPT based predictions were based on the same conversion factor and at this site on average gave predicted settlements that were within 1% of

actual values. Although not covered here it is of interest to note that for all 14 soft soil sites this method on average underestimated settlements by 6%.

6 CONCLUDING REMARKS

It can be seen from the monitoring of soft soils discussed in this paper that the prediction of behaviour is as much art as science. As it happens, the predictions of settlement magnitude and time rate compare well with the observations. This is largely due to experience based assessment of laboratory and field data together with comprehensive cone penetrometer soundings to provide the essential detailed stratigraphy.

The performance of vertical drains was as expected with settlements generally occurring rapidly. Barron's theory of radial drainage is therefore considered quite appropriate together with the smear parameters chosen. The design approach for stone columns gave good results with settlement times being similar to those for vertical drains.

The Asoaka method is recommended for pre-completion predictions of settlement since it facilitates timely decisions regarding construction programming.

Bachelier's method for estimating the magnitude of settlement provided predictions which compared well with actual behaviour. On a cost comparative basis this method provide predictions that were at least as good as laboratory based ones. The advantage of cone data is undeniable and the method deserves consideration but should be more broadly assessed before being used in isolation.

The intrigue of soft soil behaviour is highlighted in the unexpected, rapid and excessive creeping of the embankment at the East-West Arterial. Coupled with slowly decaying pore pressures this phenomenon is not unique, but appears to be very difficult to predict. In the words of Mitchell (1984) "It is essential to expect the unexpected and to deal with soils as they are, not as we might wish them to be". Needless to say, the behaviour at East-West will be monitored closely, and together with the result of subsequent investigations will be reported in greater detail at a later date. There is a clear need for further research to enable the prediction of such creep behaviour and strength gain with slowly decaying pore pressures.

7 ACKNOWLEDGEMENTS

The authors acknowledge the efforts of staff from Main Roads Queensland and Hollingsworth Consultants for data gathering and processing, and Ian Reeves (Engineer Geomechanical Services) for his review of this manuscript. They also appreciate the permission of the Commissioner of Main Roads to publish this paper. The views presented in the paper are solely those of the authors.

8 REFERENCES

Asaoka A. and Suzuki M. (1979) Settlement Predictions of Extensive Reclaimed Land. Proc. 3rd Int. Conf. on Applications of Statistics and Probability in Soil and Structure Eng., Sydney, pp. 477-486.

Bachelier M. and Perez L. (1965) Contribution a l'etude de la compressibilite des sols a l'aide du penetrometre a cone. Proc. 6th Int. Conf. SM & FE, Montreal, 2: 3-7.

Balaam N.P. and Booker J.R. (1985) Effect of Stone Column Yield on Settlement of Rigid Foundations in Stabilised Clay. Int. J. Numerical and Analytical Methods in Geomechanics, Vol. 9, pp. 331-351.

Barron R.A. (1948) Consolidation of fine-grained soils by drain wells. Trans. ASCE, Vd. 113: pp. 718-742.

Brawner C.O. and Lea N. (1963) Highway Design and Construction over Peat Deposits in British Columbia. Highway Research Record Vol. 7 pp. 1-32.

Brunsdon D. and Prior D.B. (eds.) (1984) Slope Instability. John Wiley.

Buisman A.S.K. (1936) Results of long duration settlement tests. 1st ICSMFE, Rotterdam, Vol. 1, pp. 100-106.

Crozier M.J. and Eyles R.J. (1980) Assessing the Probability of Rapid Mass Movement, 3rd ANZ Conf. on Geomechanics, Wellington, Vol. 1, pp. 47-51.

Gray D.H. and Leiser A.T. (1982) Biotechnical slope protection and erosion control, Van Nostrand Reinhold, New York.

Mitchell J.K. (1984) Practical Problems from Surprising Soil Behaviour. The 12th Karl Terzaghi Lecture, J. Geot. Eng. Divn. ASCE, Vol. 112, GT.3, March, pp. 259-289.

Parry R.H.G. (1971) Stress-Strain Behaviour of Soils. Proc. Roscoe Memorial Sym., pub. (1972) G.T. Foulis & Co. Ltd., Oxfordshire, pp. 643-668.

Parry R.H.G., Poulos H.G. and Williams D.J. (1985) Investigation, Analysis and Treatment of Soft Clay Sites, Workshop Notes, pub., University of Queensland, Sept.

Peck R.B., Hanson W.E. and Thornburn T.H. (1953) Foundation Engineering John Wiley N.Y.

Reeves I.N. (1987) Personal Communication on MRQ Files, 2 November 1987.

Richart F.E. (1959) Review of the Theories for Sand Drains. Trans. ASCE, 124, paper 2999, pp. 709-736.

Robertson N.F. (1984) The Use of Settlement Monitoring for Roadworks Construction Control, 4th ANZ Conf., on Geom, Perth, pub. Inst. Eng. Aust., pp. 321-388.

Robertson N.F. and Semple W.J. (1984) The Design and Construction of a Bridge Approach Embankment Subject to Large Settlement. Proc. 12th ARRB Conference, Hobart. Vol. 12, Part 2, pp. 103-111.

Schuster R.L. and Krizek R.H. (eds.) (1978) Landslides Analysis and Control, National Academy of Sciences, Washington D.C., Special Report 176, 240p.

Seigel R.A. (1983) Stabl User Manual, Joint Highway Research Project C-36-36K, File 6-14-11, pub. Purdue University, Indiana.

Simmons J.V. (1987) Predictability of Embankment Settlement Performance from Laboratory Consolidation Tests. Proc. 5th ANZ Conf. on Geom. Sydney (In Print).

Smith I.K. (1986) Stability of Landslides and Cuttings, A Brief Review of Geomorphic and Engineering Methods and Examination of Their Applicability to Some Additional Case Histories, M.Sc. (Hons) Thesis, Macquarie University, March 1986, 172p.

Smith I.K. (1987) Geotechnical Aspects of Marina and Waterway Developments, I.E.A. Southern Eng. Conf. Gold Coast, Aug.

Tomlinson M.J. (1963) Foundation Design and Construction, pub. Pitman, London p. 119.

Walker L.K. (1969) Secondary Compression in The Shear of Clays. J. Soil Mech. Found. Divn., ASCE, Vol. 95, pp. 167-188.

Waterton C.A. and Foulsham D.A. (1984) The Design, Construction and Performance of a Road Embankment founded on Stone Columns. 4th ANZ Conf. on Geomechanics, Perth 14-18 May, pp. 351-356.

NOTATION

ϕ	=	Angle of Internal Friction (degrees)
m_v/m_v	=	Coefficient of Compressibility (kPa ⁻¹)
c_u	=	Undrained Shear Strength (kPa)
q_c	=	Cone Tip Resistance (MPa, kPa)
s_u	=	Undrained Vane Shear Strength (kPa)
M/C	=	Moisture Content % or fractional
S	=	Soil Sensitivity = Peak Shear Strength/Residual Shear Strength
LL	=	Liquid Limit %
PL	=	Plastic Limit %
γ	=	Soil Dry Density (kN/m ³)
S_c	=	Primary Consolidation Settlement
C_x	=	Coefficient of Secondary Compression
C_v	=	Coefficient of Consolidation (cm ² /sec)
S_t	=	Settlement at time t (mm)
$S(t + \Delta t)$	=	Settlement at time (t + t Δ t) in (mm)
P_{ov}	=	Average Effective Stress kPa